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CONFINEMENT REINFORCEMENT FOR BRIDGES IN MEDIUM TO HIGH SEISMICITY ZONE BASED ON NEW CSA A23.3-04 APPROACH

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Abstract

Recent advances in confinement reinforcement of building columns have resulted in changes in Canadian code for Design of Concrete Structures CSA A23.3-04. Bridge columns and piers may also take advantage of these advances. The purpose of this paper is to use a comparable approach to propose new equations to be introduced in future Canadian bridge design code.

The adopted approach for transverse reinforcement is based on the recently developed uniaxial confinement model for concrete column at Sherbrooke University. Parametric studies have been carried out on some typical bridge columns and piers to develop equations for confinement reinforcement. An intermediate level of ductility (moderate ductility) for bridge columns and piers has been introduced, similar to that in CSA A23.3-04 building design code. Confinement reinforcement for this level of ductility has been found to be less stringent than that for ductile level. This level of ductility is suitable for regions of low to medium seismicity. The adopted approach is supported by experimental results and will provide the designer more flexibility but economical and safer seismic design of bridge columns and piers.

1. Introduction

Over the last two decades, after several damaging earthquake events, there seems to be an agreement to design structures with predictable seismic performance. However, performance based seismic design requires reliable methods to design structures to ensure that specified seismic performance goals are met. Bridges often rely solely on the capacity of columns or piers to sustain large displacements without collapsing. While

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design for specified flexural performance of reinforced concrete bridge columns or piers has become simpler nowadays, a rational approach for confinement is still needed.

The confinement requirements specified in Canadian and American bridge design codes [1,2] provide uniform confinement reinforcement regardless of ductility demand (see Section 2). When concrete strength is increased, the amount of confinement reinforcement has to be increased to reach a constant level of ductility for columns subjected to the same level of axial load [3]. Moreover, for columns or piers subjected to high level of axial load, large amount of confinement reinforcement may be needed to achieve the required ductility level. This high amount of lateral steel results in congestion of reinforcing cages and creates concreting problems. It has been suggested to increase the yield strength of transverse reinforcement to lower the amount of transverse reinforcement. However, increasing the yield strength of confinement steel does not necessarily result in increased ductility when lateral strength is kept constant [4]. Hence, there is an urgent need to revise the confinement reinforcement equations in design codes and to develop a new set of equations that will rationally take into account the effect of axial load ratio and the ductility demand.

The recently developed uniaxial confinement model [5] for concrete column at Sherbrooke University is based on strain compatibility and transverse force equilibrium and is validated with large number of experimental results. The model is capable of predicting the effectiveness of transverse reinforcement and is considered most suitable compared to other models [6]. The model has been used to develop the new equations proposed for confinement of building columns in CSA A23.3-04 [7]. Studies have shown that the new equations perform very well when compared with the available experimental results [3].

The equations developed for building columns can not be directly applicable to bridge columns or piers, as the geometry and axial load level of bridge columns or piers are significantly different. Typically, bridge columns or piers are of larger cross-section, axial load ratio is usually small, concrete cover is larger, and circular columns are widely used. Hence, a new set of equations can be developed for bridge columns and piers that consider typical construction practices. The objective of this paper is to propose new confinement equations based on two levels of ductility demand (moderate ductility and ductile) for the bridge columns and piers for inclusion in the design codes.

2. Code specifications for reinforcement details

According to Canadian and American bridge design codes [1,2], a vertical support is considered as column if the ratio of the clear height to the maximum plan dimension of the support is equal to or greater than 2.5. Supports with a ratio of clear height to maximum plan dimension of less than 2.5 are considered as wall-type piers.

Longitudinal reinforcement between 1-6% of the gross concrete cross-section area (A_g) is permitted for bridge columns in seismic performance zones 3 and 4. In Canadian code, the maximum center-to-center spacing of the longitudinal bar is 200 mm.

According to Canadian code [1], for a circular column in seismic performance zones 2, 3 and 4, the ratio of the spiral reinforcement at plastic hinge region (ρ_s) shall be taken as the larger of the values calculated from the following two equations:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_y}\left(0.5 + \frac{1.25P_f}{\phi_c f_c' A_g}\right) \tag{1}$$

and

$$\rho_s = 0.12\frac{f_c'}{f_y}\left(0.5 + \frac{1.25P_f}{\phi_c f_c' A_g}\right) \text{ where } \left(0.5 + \frac{1.25P_f}{\phi_c f_c' A_g}\right) \geq 1.0 \tag{2}$$

where, A_g is the gross cross-sectional area; A_c is the area of the core of spirally-reinforced compression member measured out-to-out of spirals; f_c' is specified compressive strength of concrete; f_y is yield strength of reinforcing bar; P_f is factored axial load at a section at the ultimate limit state; and ϕ_c is resistance factor of concrete.

American code [2] also specifies similar criteria, but it does not consider the part of the equation that takes into account the effect of axial load. However, the effect of axial load starts at $P_f/(f_c' A_g) > 0.24$, which may be rare for seismically designed bridge columns or piers.

For rectangular columns, according to Canadian code [1], in seismic performance zones 2, 3 and 4, the total cross-sectional area (A_{sh}) of transverse reinforcement at the plastic hinge region shall be taken as the larger of the values calculated from the following two equations:

$$A_{sh} = 0.30 s h_c \frac{f_c'}{f_y}\left(\frac{A_g}{A_c} - 1\right) \tag{3}$$

and

$$A_{sh} = 0.12 s h_c \frac{f_c'}{f_y}\left(0.5 + \frac{1.25P_f}{\phi_c f_c' A_g}\right) \text{ where } \left(0.5 + \frac{1.25P_f}{\phi_c f_c' A_g}\right) \geq 1.0 \tag{4}$$

where, s is the vertical spacing of transverse reinforcement; h_c is the core dimension of a tied column in the direction under consideration; and A_{sh} is the total cross-section of tie reinforcement. Similar to the requirements for circular column, for rectangular columns, American code does not consider the part of the equations that takes into account the effect of axial loads.

According to the Canadian code [1], the center-to-center spacing of transverse reinforcement at plastic hinge region shall be less than: (i) 0.25 times the minimum component dimension; (ii) 6 times diameter of the longitudinal reinforcement; or

(iii) 150 mm. However, according to American code [2], the center-to-center spacing shall not exceed one quarter of the minimum member dimension or 100 mm.

A wall-type pier may be designed as a wall-type pier in its strong direction and a column in its weak direction according to both Canadian and American code [1,2]. If the wall-type pier is not designed as a column in its weak direction, then the limitations for shear resistance is applicable as in strong direction. The reinforcement ratio, both horizontally (ρ_h) and vertically (ρ_v), in any wall-type pier is not less than 0.0025, and ρ_v is not less than ρ_h. Reinforcement spacing, either horizontal or vertically, is less than 450 mm. The reinforcement required for shear is continuous and uniformly distributed.

3. Modeling of reinforced concrete structures

3.1 Numerical simulation

Considerable amount of experimental results are available for columns subjected to constant axial load and reversed flexure. These results have pointed out the limitations of the code based design equations for confinement. However, these experimental investigations did not examine all influencing parameters in a systematic way since tests on real size columns are expensive and difficult to perform. Hence, numerical simulations can be performed for the development of the confinement equations as is done for New-Zealand standard [8]. However to be meaningful, numerical simulations shall be based on sound models reflecting the true behaviour of materials. Hence, constitutive laws of materials need to be selected carefully.

3.2 Characteristics of materials

The model proposed by Legeron and Paultre [5] can be used for the uniaxial compression behaviour of confined concrete. The model was validated with a large number of experimental results on columns made of concrete having strength 30-120 MPa confined with steel having strength 250-1400 MPa. Legeron and Paultre [5] model relates the increase of strength and ductility of concrete to the effective confinement index (I_e):

$$I_e = \frac{f'_{le}}{f'_c} \tag{5}$$

where f'_{le} is the effective confinement pressure at peak, which is a measure of the restrain applied by the stirrups to the expansion of the confined concrete core under compression and can be calculated as (see Ref. 5):

$$f'_{le} = K_e \frac{A_{shy} f'_h}{c_y s} \text{ for rectangular column in y-direction, and} \tag{6}$$

$$f'_{le} = K_e \rho_s f'_h \text{ for circular columns} \tag{7}$$

where K_e is the geometric confinement of effectiveness; A_{shy} is the total cross-sectional area of confinement steel; c_y is the cross-sectional dimension in y-direction; s is the spacing between ties; ρ_s is the volumetric ratio of spiral reinforcement to the total volume of the core; and f_h' is the stress in the confinement steel at peak stress.

The Ramberg-Osgood stress-strain curve is used to model the behaviour of the longitudinal reinforcement (see Ref. 3 for details).

Paultre and Legeron [3] analyzed the sectional behaviour of a large number of columns, considering similar material properties as above, using computer software MNPHI [9]. They have correlated ductility demand (μ_ϕ), based on the parametric studies, to the effective confinement index I_e and the axial load level n ($= N / A_g f_c'$):

$$I_e = 0.0115 n \mu_\phi \tag{8}$$

They have also found that concrete strength, volumetric ratio of longitudinal reinforcement, yield strength of reinforcement, and size of the column have only moderate influence on ductility. The most important parameter controlling ductility has been found to be the effective confinement index and the relative axial load.

4. Parametric study

4.1 Methodology

Typically, structures are designed for elastic seismic force divided by the response modification factor (R) to account for the overall ultimate capacity of the structure or force resulting from capacity design. Local ductility contributes to overall ultimate capacity and is ensured primarily by specifying the spacing and amount of sufficient confinement reinforcement. This section aims to develop design equations to obtain transverse reinforcement (A_{sh} or ρ_s) for two different levels of ductility: (i) moderate ductility level corresponding to μ_ϕ =10; and (ii) ductile level corresponding to μ_ϕ=16.
Equation (8) can be rewritten as

$$I_e' = 0.115n \text{ for moderate ductility level} \tag{9}$$

$$I_e' = 0.184n \text{ for ductile level} \tag{10}$$

Hence, substituting these effective confinement index values in Equation (5) and further substituting the values obtained in Equations (6-7), the confinement reinforcement can be calculated as:

$$A_{shy} = \frac{0.115 n c_y s f_c'}{K_e f_h'} \quad \text{for rectangular column } (\mu_\phi = 10) \tag{11}$$

$$A_{shy} = \frac{0.184 n c_y s f_c'}{K_e f_h'} \quad \text{for rectangular column } (\mu_\phi = 16) \tag{12}$$

$$\rho_s = \frac{0.23 n f_c'}{K_e f_c'} \qquad \text{for circular columns } (\mu_\phi = 10) \qquad (13)$$

$$\rho_s = \frac{0.368 n f_c'}{K_e f_c'} \qquad \text{for circular columns } (\mu_\phi = 16) \qquad (14)$$

Equations (11-14) can be easily introduced in the code base formulation for confinement reinforcement. However, K_e and f_h' must be expressed in a simple manner. Code specified minimum longitudinal reinforcement and maximum permitted spacing for longitudinal and transverse reinforcement, including seismic (Section 2) and non-seismic design considerations [1,2], are used to estimate the conservative values of K_e and f_h'.

4.2 Column and pier specimen

Parametric studies have been conducted on 600mm and 1000 mm diameter circular columns; 600 mm and 900 mm deep rectangular columns; and 600 mm and 900 deep wall type piers. Axial load ratios have been selected as 0.05, 0.1, 0.15, 0.2 and 0.3. Concrete compressive strengths have been taken as 30, 40, 50, 60, 80 and 100 MPa. The total number of columns included in this parametric study is 360 nos.

4.3 Calculation of geometric coefficient of effectiveness K_e

Calculation of K_e has been performed for all columns using the code specified minimum longitudinal reinforcement and maximum permitted spacing for longitudinal and transverse reinforcement. K_e values of 0.85, 0.80 and 0.65 have been found conservative for circular columns, rectangular columns and wall type piers, respectively.

4.4 Calculation of effective stress in confinement steel f_h'

Legeron and Paultre [5] observed that the more a column is confined the more it is able to effectively use the yield strength of the transverse reinforcement. This has also been confirmed with the experimental investigations. They suggested a procedure to calculate the effective stress in confinement steel (f_h'). The values of f_h' have been calculated for the 360 columns and piers according to the procedure suggested by Legeron and Paultre [5]. It has been observed that the values of f_h' vary from 170 MPa to f_{yh}.

4.5 Proposal for new confinement equations

Based on the result obtained from the above described parametric study the following confinement equations have been proposed:

$$\rho_s = 0.48 \frac{f_c'}{f_y} n \qquad \text{(circular columns: moderate ductility)} \qquad (15)$$

$$\rho_s = 0.54 \frac{f_c'}{f_y} n \qquad \text{(circular columns: ductile)} \qquad (16)$$

$$A_{sh} = 0.23cs\frac{f_c'}{f_y}n \quad \text{(rectangular columns: moderate ductility)} \tag{17}$$

$$A_{sh} = 0.30cs\frac{f_c'}{f_y}n \quad \text{(rectangular columns: ductile)} \tag{18}$$

$$A_{sh} = 0.27cs\frac{f_c'}{f_y}n \quad \text{(wall type piers: moderate ductility)} \tag{19}$$

$$A_{sh} = 0.43cs\frac{f_c'}{f_y}n \quad \text{(wall type piers: ductile)} \tag{20}$$

The above proposed confinement reinforcement equations (Equations 15-20) are based on ductility demand on columns and wall type piers. Hence the equations should be used in conjunction with other requirements specified in codes. For ductile wall types of piers, the response modification factor (*R*) should be taken comparable to that of rectangular columns. As the ductility demand on wall type of piers is typically low, confinement reinforcement for wall-type piers can be designed with Equation (19).

4.6 Comparison with Canadian code

Figure 1 compares the proposed quantity of confinement reinforcement to the confinement reinforcement required to reach target ductility for all the 360 columns used in parametric study. It can be observed that in most cases, Canadian code require greater quantity of confinement reinforcement. It must be noted that design codes do not specify confinement reinforcement based on ductility demand as explained in Section 2. However, from figure 1, it is evident that higher ductility demand will require more confining reinforcement. Moreover, concrete strength plays an important role in identifying the minimum reinforcement requirements. The proposed equations for confinement reinforcement present the best correlation with the results of the parametric study. In some cases, the proposed equations may underestimate the confinement reinforcement, however this underestimation may be considered reasonable for simplified equations.

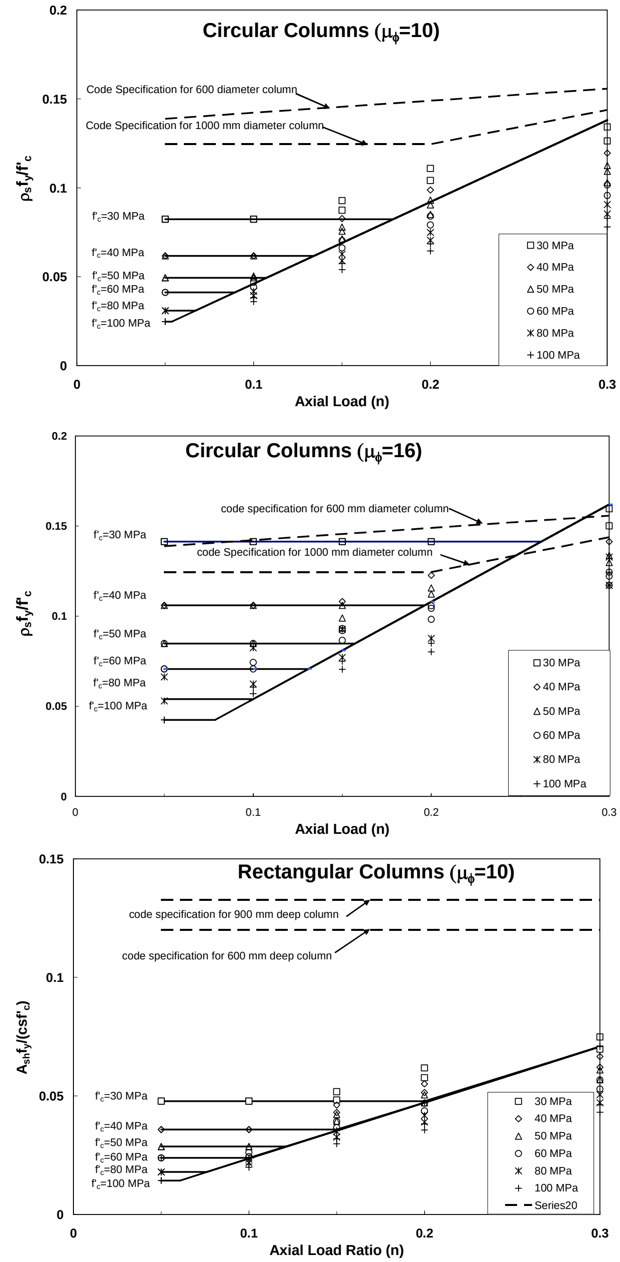

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Figure 1: validation of proposed equations for confinement reinforcement

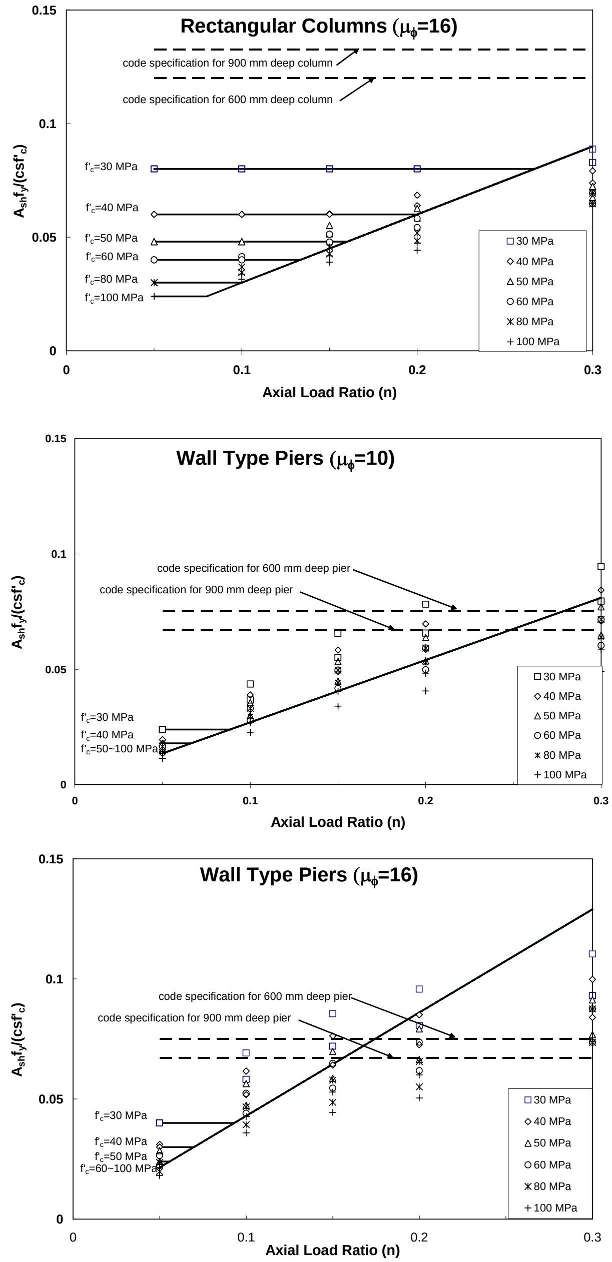

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Figure 1: validation of proposed equations for confinement reinforcement

5. Conclusions

The proposed equations for transverse reinforcement of columns and piers provide more economical and safer design and can be considered as significant improvement over the current Canadian and American Design codes. The prime advantage of the equations is that they take into the account the level of ductility and the level of axial load. Hence, these equations can be incorporated in future seismic design codes for bridges.

An intermediate level of ductility (moderate ductility) for bridge columns and piers has been introduced. The seismic response modification factor (R) equal to 2.0 for this level of ductility may seem reasonable. Confinement reinforcement for this level of ductility has been found to be less stringent than that for ductile level.

The overall approach presented in this paper can be used in performance based and displacement based design of bridges. It is believed that the next generation seismic design code will evolve toward the performance based design and hence the present work will provide a basis for more rational design of bridge columns and piers. However, it should be noted that this approach is only targeted to confinement reinforcement. Other factors that are capable of altering the experimental behaviour namely buckling of longitudinal bars and insufficient shear strength have not been considered and are the subjects of further research.

6. References

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